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Analysis of existing steel railway bridges

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Abstract

Railway bridges represent the significant parts of the railways due to their key and strategy position in transportation infrastructure. During the years of exploitation, many degradation processes and external influences attack the bridge structures. Due to those effects their durability and reliability is decreasing depending on time. On the other hand, the traffic load remains almost the same or even higher than in the past. But, bridges should not to become the limiting component of communication capacity and traffic reliability passage. In the period of 2013-14 the collective of the Department of structures and bridges has worked up the Guideline "Determination of load-carrying capacities of railway bridges" [1]. The Guideline was prepared for Slovak Railways and it was also published in the Czech Republic as the methodology instruction [2]. Therefore, the paper describes general concept and basic assumptions for evaluations of existing railway bridges and determining their load-carrying capacities based on the principles of Eurocodes.

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Keywords: Evaluation, existing railway bridge, bridge load-carrying capacity, passage of service load, partial safety factors for loads and resistances

1. Introduction

Since 1960s, a great attention was paid to initiate standardized bridge inspection, evaluation procedures and maintenance strategy to ensure sustainability of railway bridges as the important transportation infrastructure elements. Data collected through these inspection activities formed the basis for computer-based Bridge Management Systems (BMS) [3, 4, 5].

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Evaluation of existing bridges represents the substantial process and source of information relevant from the viewpoint of decision-making processes related to the strategy of bridge maintenance, repair or reconstruction [5]. To be objective, the bridge evaluation should be based on the reliability concept respecting the load-carrying capacity as the basic parameter of the existing bridge reliability. Therefore, evaluation of existing bridge should incorporate not only periodic inspection and subjective assessment of actual bridge condition but also verifying the bridge reliability affected by the actual bridge condition.

From this viewpoint, the bridge evaluation should be considered when significant deviations from the project descriptions are found, when some relevant damage is observed, or when the bridge exceeds its planned service life. Concurrently, the bridge load-carrying capacity is the decision parameter for determining the passage of corresponding railway service load over the bridge.

2. Generally

The below described concept of the determination of load-carrying capacities of existing bridges is fully compatible with the principles of Eurocodes. The Guideline follows the Service handle SR5 [6], valid till publishing the new ones. The structure of the Guideline is consistent with European standards. Firstly, the general parts valid for all types of bridges are introduced. Then, the four Annexes (from A to D) specify the rules for determination of load-carrying capacities of individual bridges according to the materials, of which they are manufactured, i.e. steel bridges, composite steel and concrete bridges, concrete bridges, and masonry bridges, respectively. The Annex E introduces a pattern for the table of summarized load-carrying capacities of bridge members and parts. In the Annex F, the approaches to more precise calculations of the partial safety factors for load effects and cross-sectional and structural member resistances are given.

Two types of the bridge load-carrying capacities are introduced in the Guideline. Normal load-carrying capacity (further load-carrying capacity only) is defined as a dimensionless quantity expressing the ration of the limit effects of variable vertical rail traffic load from the viewpoint of satisfying relevant ultimate or serviceability limit states, to the effects produced by the Load Model 71 (LM71) according to standard [7]. From this definition it is clear, that the load-carrying capacity should be expressed in the form of Rating factor (RF_{LM71}) of variable traffic load represented by the LM71. The newly introduced exceptional load-carrying capacity should be determined by means of analysis of the existing bridge according to approaches and principles presented in the Guideline respecting additional alleviations compared to normal load-carrying capacity.

Relevant bridge administrator or competent authority of Slovak Railways specifies the required category of the bridge load-carrying capacity. According to the precision of methodology used for the load-carrying capacity determination, four categories designated as A, B, C and D are defined in the Guideline. The great attention is paid to the determination of load-carrying capacities of category C and D, which are defined as follows:

- Category C: the load-carrying capacity determined by means of analysis of existing bridge based on its verified actual condition or in the case of a new bridge based on the results of its design analysis
- Category D: the same procedure as for category C has to be done and completed by the experimental analysis of the bridge behavior verifying the correctness and availability of the bridge computational model

The evaluation of the passage of the appropriate service load defined in accordance with the standard [8] represents the relevant result of the determination of the bridge load-carrying capacity. The service load is the general term for line and locomotive categories of railway traffic load with associated speeds and also for special railway track vehicles.

The general part of the Guideline specifies background and relevant principles and rules valid for all types of railway bridges to determine their load-carrying capacities based on the fulfilment of the appropriate criteria of the relevant ultimate and serviceability limit states. It means specification for bridge actions, dynamic effects of the variable traffic loads, partial safety factors for actions, material properties and procedures of determining its design values, methods of the bridge global analyses and procedures for determination of bridge load-carrying capacity depending on its category mentioned above (A, B, C, and D). The procedure how to evaluate the passage of the service load over the assessed bridge based on the comparison of the determined load-carrying capacity to the efficiency of the service load represented by the relevant line categories is also presented in the general part of the Guideline.

2.1. Actions on existing bridges

All main provisions according to Eurocode for actions on bridges [9] should be respected. The current state of the location and the value of actions should be identified by in-situ measurement compared to available bridge documentation. Characteristic values of the permanent loads on existing bridges should be taken according to standard [9] respecting Annex D in the standard [10] and code [12] in cases, when the results of experimental investigations on the bridge are taken into account. Densities of materials are determined indirectly on the basis of standard values or information from tables or, in appropriate cases, directly from specimens or samples taken from bridge.

For existing bridges, the vertical rail traffic load is represented by the LM71 in accordance with standard [9] using $\alpha = 1.00$. The horizontal forces multiplied by $\alpha = 1.00$ (nosing force, centrifugal force, braking and acceleration forces), which should be considered together with the vertical rail traffic load on existing bridges, are also defined according to the same design code [9]. Dynamic effects of the vertical rail traffic load should be taken into account by means of the dynamic factors Φ_2 or Φ_3, which shall be determined according to code [9]. Unless the relevant authority of the railways specifies the dynamic factor, factor Φ_3 shall be used for determination of the existing bridge load-carrying capacity. Regardless of bridge deck type, redistribution of the vertical load to the three rail support close to the acting axle forces of LM71 or other concentrated wheel loads may be assumed for the load-carrying capacity determination. This assumption is also valid for redistribution of the horizontal forces. The effects of the eccentricity of railway track on the bridge deck shall be taken into account by means of measurement of the actual track location on the bridge. Besides the method given in the figure 6.4 in the standard [9], the redistribution can be modeled using more precise assumption taking into account the actual layout of rail supports including the stiffness of sleepers and ballast. Eccentricity of vertical load according to figure 6.3 in the standard [9] may be neglected when considering load-carrying capacity determination. Accidental actions should also be considered in accordance with the design code [9]. Bridge structural members shall be assessed on the accidental load effects but the load-carrying capacity is not required to be determined for this design situation.

The reliability of existing bridge structural members and their cross-sections shall be verified and their load-carrying capacity shall be determined using partial safety factors method. The partial factors of load effects are used to determine design values of permanent (self-weight of structural and non-structural bridge elements), variable load effects on existing bridges (LM71, nosing force, centrifugal force, braking and acceleration forces) and design values of non-traffic load effects due to wind and thermal loads. The following values of partial factor γ_G should be considered for determining the design values of permanent load effects on existing bridges, unless more precise calculation is used:

- Structural members of existing bridges whose geometrical parameters were checked by measurements: $\gamma_G = 1.20$
- Structural members of existing bridges whose geometrical parameters were not checked: $\gamma_G = 1.30$

Partial factor $\gamma_{Q,LM71}$ for the vertical variable rail traffic load effects caused by LM71 should be considered in dependence on the age of bridge structural member and its planned remaining lifetime using following values:

- Bridge structural member younger than 30 years: $\gamma_{Q,LM71} = 1.40$
- Bridge structural members older than 30 years: $\gamma_{Q,LM71} = 1.25$

The partial factors for wind and thermal load effects should be taken by the following values in dependence on the age of bridge structural member and its planned remaining lifetime:

- For bridge structural members younger than 30 years: $\gamma_Q = 1.50$
- For bridge structural members older than 30 years: $\gamma_Q = 1.35$

The values of partial factors for permanent and variable rail traffic load effects were derived from the values valid for design of new bridges, which are defined in the National Annex [15]. Procedures described in paper [13] were used to derive values of partial factors for load effects valid for existing bridge evaluation. Partial factors for the load effects concerning the serviceability limit states should be considered by values of $\gamma_{F,ser}$ = 1.0. The Guideline provides possibility to determine values of partial factors for load effects by means of the more precise procedure according to articles [13], [14] given in Annex F of this Guideline in dependence on the age of bridge structural member and bridge planned remaining lifetime, which should be defined by the relevant authority of the Slovak Railways.

3. Analysis of existing steel railway bridges

3.1. Material properties

Material prosperities and their design values should be determined either:

a) On the basis of the by-inspection verified documentation and standards valid at the time of bridge design, or
b) On the results of diagnostic techniques and material tests.

In the case of a), the guaranteed yield strength given in bridge documentation or in the standard valid in time of bridge design may be considered as the characteristic value of steel yield and ultimate tensile strengths. Those values should be verified using hardness testing methods. The design cross-sectional and member resistances for ultimate limit state (without fatigue) verifications should be determined using values of design yield and ultimate strengths obtained by dividing the characteristic values by partial factor γ_{Mi}, which may be taken from Table 1. If the construction year of the bridge is known and no doubts are about the material classification (S235, S275 and S355), then Table 1 may be used to estimate the characteristic and design values of steel yield and ultimate strengths. Partial factors for design cross-sectional and member resistances concerning the serviceability limit states should be considered by the values $\gamma_{M,ser}$ = 1.0.

Unless bridge documentation is available or there are doubts about the quality of the material used, procedures in accordance with b) should be applied. In this case, the properties of steel material shall be determined by tests according to standards [10], [11] or relevant provisions in Guideline. The values of partial factors for resistances γ_{Mi} should be calculated using the more precise approach (see standards [10], [11]). Unless more precise methods are considered to determine the values of partial factors, the values from Table 1 may be used or the procedures given in Annex F of the Guideline may be applied using statistical parameters of steel obtained by tests.

3.2. Global analysis

The spatial computational models should be used for global analyses of steel railway bridges to enable the more precise approximation of the actual bridge behavior and allowing for the effects of the possible imperfections or damages of the structural members and bridge parts. Internal forces and moments should be determined using elastic method. Usually, the elastic first order method is recommended to apply using the initial structural shape with respect of the relevant criterion for application of the first order theory (see standard [16]). Actual bridge structural member's condition shall be respected in global analysis of existing bridge structure. Possible failures and damages including relevant imperfections of structural members or bridge parts should be involved into the appropriate computational model to take into account influence of the failures on the final bridge response to load respecting the load redistribution due to failures. Concurrently, the influence of the bridge damages shall be taken into account in the bridge structural member and cross-sectional resistances. The shape and size of imperfection significantly affecting the member load-carrying capacity is recommended to be obtained by means of measurements on the real bridge structure, especially in the case of riveted bridge structural elements where the effects of residual stresses is negligible.

Table 1. Recommended material characteristics of steel and values of partial factors for cross-sectional resistances and member

Year of bridge construction	Steel material/ grade	Thickness [mm]	Allowable stress σ_{adm} [MPa]	Yield strength f_y [MPa]	Ultimate strength f_u [MPa]	γ_{M0}	γ_{M1}	γ_{M2}
before 1895	wrought iron		130	210	340	1,10	1,20	1,30
1895-1904	wrought iron		130	210	340	1,10	1,20	1,30
	mild steel		140	230	360	1,10	1,20	1,30
1905-1937	mild steel		140	230	360	1,10	1,20	1,30
1938-1950	37 (S235)		140	230	360	1,10	1,20	1,30
	52 (S355)		195	335	490	1,10	1,25	1,30
1951-1968	37 (S235)	t ≤ 25	140	230	360	1,10	1,20	1,30
		> 25	130	210	340	1,10	1,20	1,30
	52 (S355)	≤ 16	210	360	510	1,10	1,25	1,30
		> 16	200	340	490	1,10	1,25	1,30
1969-1985	37 (S235)	≤ 25		235	360	1,00	1,10	1,25
		> 25		215	360			
	52 (S355)	≤ 50		355	510			
1986-1998	37 (S235)	≤ 25		235	360	1,00	1,10	1,25
		> 25		215	360			
	52 (S355)	≤ 25		355	510			
		> 25		335	470			
1998-2010	S235	≤ 40		235	360	1,00	1,10	1,25
	S235	40<t ≤ 80		215	360			
	S275	≤ 40		275	430			
	S275	40<t ≤ 80		255	410			
	S355	≤ 40		355	510			
	S355	40<t ≤ 80		335	470			

Among the main representative imperfections and member's damages, the following ones should be considered in the global analysis:

- Noticeable global buckling of the member systems creating one unit (arch bridges, compression chord of truss bridges openly arranged)
- Significant deformations of the structural members and parts of bridge steel structure due to vehicle impacts
- Absenting members or bridge parts
- Significantly corroded cross-sections of structural members

In the case of bridges with welded cross-sections, the equivalent geometrical imperfections according to standards [16], [17] may be used in the bridge global analysis. In this case, the assumed imperfection shape of the steel bridge structural member or part is recommended to be derived from the shape of elastic critical buckling mode of a structure as a unique global and local imperfection. The amplitude of this imperfection may be determined from 5.3.2 in standard [16]. In the case of plated structures, the shapes and amplitudes of equivalent geometrical imperfections may be determined in accordance with Annex C in standard [17]. Local imperfections of cross-sections and members may be also allowed for into their resistances using relevant reduction factors for flexural, lateral-torsional or plate buckling using method of equivalent member according to standard [16].

In global analysis of bridge plated structures, the effect of shear lag shall be taken into account. Unless more precise approach is used, the effect of this phenomenon may be taken into account by means of an effective width. The effect of plate buckling in the elastic global analysis may be taken into account by effective cross-sectional areas of the elements in compression. This effect on the stiffness may be ignored when the effective cross-sectional area of an element in compression is larger than ρ_{lim} times the gross cross-sectional area, where ρ_{lim} = 0.5 according to standard [17].

When the load-carrying capacity of the bridge structure should be significantly limited by resistances of very slender compression member, there is a possibility to omit this member from global analysis upon reaching their compression resistance, provided that the elastic redistribution of internal forces and moments is admissible and the remaining part of the structure is allowable to carry the acting loads.

3.3. Determination of load-carrying capacity for ultimate limit states

The reliability of existing bridge elements shall be verified and their load-carrying capacities shall be determined using partial safety factors method, by which the fulfilment of conditions of relevant ultimate limit states in frame of the appropriate design situation shall be assessed.

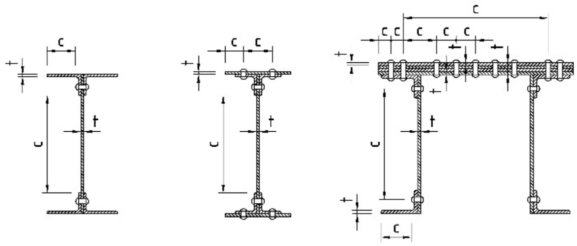

Fig. 1. Definition of widths for classification of riveted cross-sections first picture.

Welded cross-sections should be classified with respects to slenderness of their partial parts in accordance with Table 5.2 in the standard [16]. Longitudinally stiffened compression webs and flanges should be classified respecting the rules defined in the standard [17]. For classification of the riveted cross-sections, the widths of relevant cross-sectional parts defined in Fig.1 should be taken into account. Except for transversal direction, the classification should also respects the rivet distances parallel to the acting compression stress. Due to unknown behavior of riveted cross-sections in plastic area of stressing, the elastic resistance should be considered for assessment of the cross- sectional resistance.

Regarding the ultimate limit states, the load-carrying capacity of the bridge structural member should be defined by means of the general equation expressing the Rating Factor (RF_{LM71}) of variable traffic load represented by the LM71 in the following form

$$RF_{LM71} = \left(R_d - \sum_{i=1}^{n-1} E_{rs,Ed,i} \right) \Big/ E_{LM71,Ed} \text{ , where} \tag{1}$$

R_d is the cross-sectional resistance of the bridge structural member

$E_{LM71,Ed}$ represents design value of vertical variable rail traffic load effects represented by the LM71 including dynamic factors

$\Sigma E_{rs,Ed,i}$ is design, combination or group values of the other load effects acting concurrently with the vertical rail traffic load

The load-carrying capacity of cross-sections classified into the classes 1, 2 and 3 subjected to combination of bending, shear and normal force, provided that design shear force V_{Ed} fulfils the following condition

$$\eta_3 = \frac{V_{Ed}}{V_{pl,Rd}} \leq 0{,}5 \quad \text{resp. } \bar{\eta}_3 = \frac{V_{Ed}}{V_{bw,Rd}} \leq 0{,}5\,, \tag{2}$$

should be determined using equation as follows

$$RF_{LM71} = \frac{1-\eta_{1,rs}}{\eta_{1,LM71}}\text{, where} \tag{3}$$

$$\eta_{1,rs} = \frac{N_{rs,Ed}}{A \cdot f_y / \gamma_{M0}} + \frac{M_{y,rs,Ed}}{W_{el,y} \cdot f_y / \gamma_{M0}} + \frac{M_{z,rs,Ed}}{W_{el,z} \cdot f_y / \gamma_{M0}} \tag{4}$$

$$\eta_{1,LM71} = \frac{N_{LM71,Ed}}{A \cdot f_y / \gamma_{M0}} + \frac{M_{y,LM71,Ed}}{W_{el,y} \cdot f_y / \gamma_{M0}} + \frac{M_{z,LM71,Ed}}{W_{el,z} \cdot f_y / \gamma_{M0}} \tag{5}$$

$V_{pl,Rd}$	is the design plastic shear resistance
$V_{bw,Rd}$	is the design shear resistance respecting web shear buckling
$N_{LM71,Ed}$, $M_{y,LM71,Ed}$, $M_{z,LM71,Ed}$	are the design values of internal forces and moments caused by the vertical variable rail traffic load represented by LM71 including dynamic factors
$N_{rs,Ed}$, $M_{y,rs,Ed}$, $M_{z,rs,Ed}$	are the design, combination or group values of the internal forces and moments due to other load effects acting concurrently with the vertical rail traffic load
A, $W_{el,y}$, $W_{el,z}$	are the cross-sectional characteristics
γ_{M0}	is the partial safety factor for cross-sectional resistance

Because of dependence of the shear force V_{Ed} on the load-carrying capacity in equation (2), the calculation of RF_{LM71} should run using an iterative approach. An iterative calculation of load-carrying capacity should be also used when the condition in equation (2) is not fulfilled. Verifying the plastic resistance of welded cross-sections of classes 1 or 2 subjected to combination of bending, shear and normal force and their load-carrying capacities determination can be conservatively performed using equation (3) in that the plastic resistances instead elastic ones should be used in equations (4) and (5). Verifying resistances of slender cross-sections of class 4 shall respect effects of the shear lag and plate buckling, which may be taken into account by means of effective cross-sectional characteristics. Provisions given by 6.2.9.3 in the standard [16] and chapter 3 and 4 in the standard [17] should be applied. Load-carrying capacity of the slender cross-section subjected to bending, shear and normal force can be determined using equations (3) in that the effective cross-sectional characteristics should be substituted into equations (4) and (5) and possible shift of the centroid of the effective cross-sectional area A_{eff} relative to the center of gravity of the gross cross-section according to 6.2.2.5 in the standard [16] shall be allowed for.

Buckling resistance of the compression member and resistance against lateral-torsional buckling of members subjected to major axis bending should be estimated in accordance with 6.3.1 and 6.3.2 in the standard [16]. The load-carrying capacities of those members may be determined by means of the following equations

$$RF_{LM71} = \left(N_{b,Rd} - N_{rs,Ed}\right) / \mathrm{N}_{LM71,Ed}\ ,\quad RF_{LM71} = \left(M_{b,Rd} - M_{rs,Ed}\right) / \mathrm{M}_{LM71,Ed}\text{, where} \tag{6}$$

$$N_{b,Rd} = \chi A \cdot f_y / \gamma_{M1}, \quad M_{b,Rd} = \chi_{LT} W \cdot f_y / \gamma_{M1} \tag{7}$$

χ is the reduction factor for the relevant buckling mode of the flexural buckling
χ_{LT} is the reduction factor for lateral-torsional buckling

To determine the load-carrying capacity of the member subjected to compression and biaxial bending, the condition of reliability verification in accordance with 6.3.3 and Annex B valid for method 2 in the standard [16] shall be applied. The load-carrying capacity of this member should be estimated by means of iterative approach.

Other parts of the Annex A of the Guideline are devoted to specifications related to individual types of steel bridges and their decks describing principles of determination of their load-carrying capacities.

4. Conclusions

The submitted paper informs about new Guideline for determination of railway bridge load-carrying capacity. It introduces brief description of general parts of the new Guideline and also pays attention to the methodology of estimation of load-carrying capacities of steel bridge structural members and their cross-sections.

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